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# SEISMIC DESIGNED BACKSLOPES AND EVALUATION IN A STRUCTURALLY DISTURBED BASALT SECTION

A Thesis

Presented in Partial Fulfillment of the Requirement for the

Degree of Master of Science

Major in Geology

in the

University of Idaho Graduate School

by

Ronald E. Larsen

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AUTHORIZATION TO PROCEED WITH THE FINAL DRAFT:

This thesis of Ronald Edward Larsen for the Master of Science degree with major in Geology and titled "Seismic Designed Backslopes and Evaluation in a Structurally Disturbed Basalt Section," was reviewed in rough draft form by each Committee member as indicated by the signatures and dates given below and permission was granted to prepare the final copy incorporating suggestions of the Committee; permission was also given to schedule the final examination upon submission of two final copies to the Graduate School Office:

Major Professor Terry R. Howard Date 4/12/76

Committee Members B. Charboneau Date 4/28/76

_______________ Date _______

REVIEW OF FINAL DRAFT:

Department Head _______________ Date _______

College Dean _______________ Date _______

FINAL EXAMINATION: By majority vote of the candidate's Committee at the final examination held on date of _______ Committee approval and acceptance was granted.

Major Professor _______________ Date _______

GRADUATE COUNCIL FINAL APPROVAL AND ACCEPTANCE:

Graduate School Dean _______________ Date _______

# BIOGRAPHICAL DATA

Ronald Edward Larsen
September 1, 1940 - October 20, 1975

## Education

B.S. - University of Wisconsin, 1964 (Geology)

Graduate Work, University of Wisconsin 1964-1965

Graduate Work, University of Idaho 1971-1976

M.S. - University of Idaho, Posthumously, 1977 (Geological Engineering)

## Professional Employment

1967 - 1975 - Idaho Transportation Department, Lewiston, Idaho

1966 - 1967 - James Cape & Sons, Racine, Wisconsin

1966 - 1966 - John H. Nielson & Associates, Racine, Wisconsin

1965 - 1966 - Mobile Oil Company, Corpus Christi, Texas

1963 - 1965 - U.S. Steel

## Research

Columbia River Basalt

# ACKNOWLEDGEMENTS

Direction for this project was given by George W. Williams, the University of Idaho Geology Department Chairman, John G. Bond, P. G., Senior Geologist and Acting Director for the Idaho Bureau of Mines and Geology. J. W. Schumaker, former District Four Geologist, Department of Transportation, now with Walter Lum and Associates, Honolulu, Hawaii, prepared the prototype soils profile which the District has adopted as standard procedure and was the inspiration for the scale model approach. R. G. Charboneau, Chief Geologist, Idaho Transportation Department, Division of Highways, offered both guidance and criticism, primarily with geophysical techniques and preparation of the manuscript. William B. Hall, Professor of Geology, University of Idaho provided many of the preliminary aerial photos and M. W. Lotspeich, District Engineer, Idaho Transportation Department, Division of Highways who authorized and supported this paper.

This paper was edited by Robert R. Elvin and R. G. Charboneau, Idaho Transportation Department, Division of Highways.

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# SEISMIC DESIGNED BACKSLOPES AND EVALUATION IN A STRUCTURALLY DISTURBED BASALT SECTION

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Idaho Transportation Department

## ABSTRACT

Highway construction in mountainous terrain has suffered from limited or inadequate investigations concerning the engineering properties of rocks and geologic discontinuities.

Projection of surface mapped structural features and rock quality in relation to highway grade or backslope design is often proven to be in error, especially in geologically complex areas.

During the 1960's the Idaho Division of Highways formulated plans to locate, design and build, through stage construction, a modern, multi-lane highway through mountainous terrain of West-Central Idaho.

Although some diamond drill exploration was attempted, the bulk of the final design was based solely on geologic mapping and seismic refraction methods, in which overburden, depth of weathering, bedrock configuration, and structural elements were determined.

# SEISMIC DESIGNED BACKSLOPES AND EVALUATION IN A STRUCTURALLY DISTURBED BASALT SECTION

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## INTRODUCTION

The relocation of U.S. 95 in the White Bird area will replace a highway that was initially graded in 1919-1920, surfaced in 1922-1923 and finally paved in 1938. Since U.S. 95 serves as Idaho's only North-South route and link to the "Panhandle", it is of utmost importance to connect the State with an updated mode of transportation.

The new highway along the East face of White Bird Hill will eliminate 14 switchbacks (60° curves), and the "ladder back" design.

The final design phase on the first of five projects began in the summer of 1966. The final design on the fourth project was completed in June of 1971.

Also included in this report is the background and history of both the corridor selection and various alignment considerations within the chosen corridor, some of which was based on geologic criteria, while others had political overtones.

This paper is not concerned with the age of various units, the source of the basalt, or stratigraphic correlation on a regional basis, but only with the engineering properties of the basalt and associated sediments that will govern the success or failure of the final design on the White Bird Hill projects.

## MAGNITUDE, DESIGN STANDARDS AND COST

The selected corridor was divided up into five projects to be completed by stage construction.

| Contract Let | Project | | Miles | Bid Price |
|---|---|---|---|---|
| 1969 | F-4113(31) | White Bird Summit-South New Construction | 3.5 | $1.8 Million |
| 1970 | F-4113(32) | White Bird-North New Construction | 2.9 | $2.2 Million |
| 1972 | F-4113(44) | White Bird Bridge | | $1.7 Million |
| 1973 | F-4113(41) | Paving Contract | | $1.2 Million |
| 1973 | F-4113(38) | White Bird-Salmon River New Construction | 2.1 | $1.1 Million |
| | | TOTAL | 8.5 | $8.0 Million (Plus change orders and claims) |

The design standards for these projects were as follows: 60 MPH Design Speed; 34' Minimum Width; 12'-wide Truck Lane; 10' Flat Bottom Ditch; 2.0' Rock Cap; Maximum 8° Curves; and 6.8 to 7.0% Grade on (31) and (32) projects. The highway rises from an elevation of 1486' on the river to 4245' at White Bird Summit, a distance of 8.5 miles. The completion of these projects will reduce both driving time and distance by 50%.

One of the basic reasons for this price was the amount of information supplied to the contractor in the form of a soils profile. Shown on the profile is a wealth of information, both geological and geophysical, along with the recommended slope design, which is related to the subsurface, on a typical cross section every time the slope design changes. If used, the contractor may govern his pre-splitting operation from this data, as was the case on White Bird.

## STRATIGRAPHY

Stratigraphically two types of basalt will be recognized on the project, Upper and Lower Columbia River Basalt (Bond, 1963). Individual flows and flow units will be described, but no attempt will be made to equate either age, or petrographic similarities to the type Yakima or Picture Gorge sections of Washington or Oregon. The contact between Upper and Lower Basalt is recognized by the drastic break in slope and color differential of the weathered exposures. Above the contact, vertical stair-stepped or tiered outcrops, and darker colors (blue and black) predominate, whereas below the contact the slopes are more gentle and smoother and the outcrops are brown to reddish-brown in color.

If treated properly both types of basalt will provide stable back-slope conditions and good compaction within the embankments.

A detailed lithologic examination will show the following observations to hold true for the Upper Basalt:

1. The basal colonade is absent to thin, but the total flow is relatively thick.
2. Irregular, thin (average 10'' diameter) sliver columns are tightly jointed and resistant. The jointed surfaces are fresh, sharp-edged and relatively unweathered.
3. A fine uniformly grained texture persists throughout. Occasionally a few small phenocrysts are found.
4. Upon weathering the end product is clay.

The Lower Basalt usually possesses the following:

1. The basal colonades may be up to one-third the total thickness of the flow.

2. The columns which are highly variable in size are usually well developed. They may be vertical wavy or rosetted.
3. A porphyitic texture predominates with the cleavage surface having a resinous luster.
4. The prominent joints will be rounded and may easily disintegrate to a sandy end product.

BASALTS AS ENGINEERING UNITS

Basalts are unique from many standpoints. They all look alike, especially from a distance. For engineering purposes the flow concept is much too broad to be of any use except in correlation and structural analysis.

Each flow should be broken down into its individual components (units) and then examined for both physical and chemical anomalies. A close examination will point out significant changes that may occur in the same unit along strike. The most noticeable difference is related to the jointing habit or proximity to structural displacements. Venting, along with susceptibility to weathering and groundwater will also drastically affect the integrity of the unit.

The various engineering types or flow components which had to be contended with on the White Bird projects are as follows:

1. Massive - poorly defined columns, weakly jointed, dense, internally tight fractures, high seismic velocities (Platy-Conchoidal).
2. Columnar - well developed joints, medium to high seismic velocities.
3. Entablature - well developed, tight joints (Hackly Fracture) 6"-10" average diameter, high velocities.

4. Flow Top - poorly developed joints, highly indurated, low velocities.
5. Pillow - Plagonite - no joints, low velocities.
6. Flow Breccia - no joints, medium velocities.
7. Fault Breccia - highly fractured, occasionally semi-cemented, usually small fragments, low velocities.
8. Interbed - laminated bedding, fine grained, low velocities.
9. Basalt Talus - variable units of soil and cement. Medium to low velocities, dry to damp.
10. Colluvium - low velocities, dry to damp.
11. Lake Sediments - highly variable in both attitude and facies.
12. Ash Deposits - low velocities, no bedding.

Stress release in respect to backslope design was considered within the faulted area and with the use of seismic analysis was found to be nominal. The rock was simply more fractured, some of which indicated drag.

## METHODS OF SUBSURFACE EXPLORATION AND ROCK EVALUATION

Although a Mobile B-40, 5" auger and Sprague and Henwood diamond drill was used to retrieve both soil and continuous rock core, the bulk of design information was derived from geophysics, primarily seismic refraction.

A portable single channel, engineering seismograph manufactured by Geophysical Specialties, Model MD-1, was the main source of information. Correlatable rock velocities were substituted for drill holes in inaccessible areas.

A hammer struck on a metal plate supplied the energy for most traverses of 100' in length or less, except in fault zones. Explosives (Primacord, Primadent, Dynamite--40 and 60% Gel-7/8" diameter) were detonated by a battery operated blaster and seismic blasting caps.

Any explosive charge was buried at least one foot below the natural ground surface and then backfilled, preferably with soil, in order to promote vibration into the rock rather than air waves.

The state-of-the-art has progressed so rapidly within the past seven years, the new digital readout devices, signal enhancement, memory recall, increased reception and sensitivity, and with portability of the multi-channel units, one would hardly recognize the prototype.

Any future work of these magnitudes should utilize a multi-channel seismograph for deep penetration, as only one explosive charge is necessary to determine a contact and substantiate slope velocity.

A signal enhancement unit is faster for shallow work, will not exhaust the manpower, and allows the operator to visually build a first arrival sine wave.

## EXISTING LAB TESTS USED BY THE DIVISION

Presently, potential quarry rock is subjected to four lab tests which will determine the quality of a particular basalt. Three are mechanical in nature while the fourth is chemical.

The Los Angeles Abrasion Test (AASHTO T-96) (L.A. Wear), is a mechanical test in which a known amount of basalt is retained on a No. 12 sieve, placed in a revolving cylinder with 6 to 12 steel bearings and rotated for 500 turns. The material retained on the No. 12 sieve is then weighed and compared to the original value. The higher the number the less resistant the rock. Generally a value under 20 is quite acceptable and a value over 35 is considered a failure.

The Sand Equivalent Test (AASHTO T 186-70) (T-2-68) serves to show the relative proportions of fines or clay-silt particles (passing the No. 4 sieve) in graded aggregates that have already been crushed. The SE is computed by dividing the sand reading by the clay reading in a graduated cylinder and then multiplying by 100. The higher the SE, the more durable the material. A value below 30 is generally unacceptable.

The Idaho Degradation Test (T-15-72) is another mechanical abrasion test. The sample is run in a saturated surface dry condition (16 hours soaking), that measures the increased percentage of fines passing the No. 200 sieve. This serves as a quantitative measure of the resistance of a coarse and fine aggregate to the production of plastic fines.

The use of Ethylene Glycol to promote accelerated expansion of reactive aggregates is a chemical test used to determine the percentage of unaltered rock retained, after immersing a specific amount of basalt, in a Glycol solution for 15 days. The sample is then oven dried, and the amount retained on a 3/8" sieve is measured.

The Glycol acts as a chemical catalyst speeding up any potential reaction or expansion of any unstable minerals and swelling clays. Newton H. Olson, Army Corps of Engineers, Walla Walla, stated (personal communication) that rock samples, which normally take 10-12 months of atmospheric exposure to promote failure or degradation can by the Glycol test, 15-day immersion, be accelerated to rapid failure. The presence therefore of montmorillonite or swelling clays in a sample can be ascertained. Experience states that any less than 93% retained is questionable.

AREAS OF SPECIAL CONSIDERATION

Station 352 - Engineering vs. Ecologic Impact

The alignment throughout the Salmon River section of this project was in limbo for many years. The original proposal was to stay on natural ground, which consisted of a thick, truncated, colluvial terrace deposit that was interlaced with lenses of fluviatile sands. An existing cut slope along the old highway was standing at a $\frac{1}{2}$:1 showing large blocks of subangular basalt within a silt matrix, that at least on the outer weathered surface was partially cemented.

Due to the excessive height and quantities involved for the proposed cuts ($1\frac{1}{4}$:1), it was recommended to shift the alignment towards the river, which in places would mean a partial encroachment into the Salmon River.

This presented no real problem as there was a solid bedrock foundation to build on. River current velocities were recorded so that adequate sized riprap could be placed. The velocities were 12'/sec., which according to the Corp of Engineers should mean a stone weight of over 400 lbs.

Engineering-wise the problem could be solved, but environmentally there was a snag. The problem was that any river encroachment must have the sanctioned approval of the various environmental agencies, of which the Idaho Fish and Game Commission is one. They recommended that since willows were in abundance along the existing bank that two 20-foot-long jetties (10 feet wide) be constructed at right angles to the riprap facing. This would, hopefully, encourage sand deposition and a possible spawning area.

With the maximum daily discharge of 100,000 C.F.S. and stream velocities of 12 foot per second in a partially restricted narrow channel having a 90 degree bend in the river immediately upstream, the jetties would be washed out and would have to be replaced on a yearly basis. For this reason, the Materials Section was asked to review the originally proposed line. A seismic survey was initiated which consisted of ten normal-reverse contoured traverses which would give an overlap of information (stair-step coverage).

The contact depths were computed in the usual fashion and then plotted to the appropriate cross sections. To be exact, in the analysis one must be aware that sound waves will pick up a discontinuity that may not be in a horizontal or uniformly dipping plane. This is especially true in basalts which are typically tiered, weathered and covered by either talus or colluvium. For this reason, the depth was shown as an arc, rather than a point, and somewhere along that arc was a more resistant layer, which possibly represented bedrock. See Photo No. 1.

PHOTO NO. 1

M.P. 221.50
3/4:1 Colluvial Backslopes with 10' Benches every 50'

After the data was accumulated it appeared that most feasible slope designs would remain in colluvial debris. A 3/4:1 slope with 10-foot benches for every 50 feet of vertical height was selected even though the natural slope was 1½:1. This decision was based on excess quantity with a flatter slope and the fact that the existing cut slope was standing adequately on ½:1 slope. Another factor is the annual rainfall which amounts to 6''/year. No undisturbed soil samples were taken as the material was much too rocky to test, so the factor of safety is still unknown.

So far the slope has performed very well with only one small bench failure in the alluvial sand. See Photo No. 1.

Station 437 - Banner Ridge Section

A 120-foot cut on centerline was proposed through this area which would amount to considerable excavation, depending on the slope design.

There were a number of faults within the vicinity but what effect this would have on the rock quality was unknown. Three diamond drill holes were placed, but very little core was recovered. The bulk of the material was drilled by tricone.

Seismic traverses were attempted but the data appeared scrambled and there was no continuity between traverses. The only exposure was a thin Breccia unit and this was not continuous as it was dissected by at least two faults. Three more diamond drill holes were placed with similar results.

At this point it seemed foolish to attempt more work. The slopes were designed for the worst possible conditions; 1½:1 overall, serrated slopes (mini benched), with a 30-foot bench every 50 feet vertically. See Photo No. 2.

PHOTO NO. 2

M.P. 223.20
Banner Ridge - $1\frac{1}{2}$:1 Mini-bench Slopes in an Altered Basalt

The recommendation was correct, the bulk (95 percent) of the cut was steeply dipping, hydrothermally altered, and structurally displaced. See Photo No. 2. The material throughout the cut is weathering and differentially sluffing back to the $1\frac{1}{2}$:1 overall slope. Hopefully the grasses will establish their roots and eliminate raveling and look asthetically appealing.

The seismic data was useless under these circumstances, which was the reason it looked like a puzzle. The little silver box was correct, just the operator was confused.

<u>Sta. 492.00-494+50 - Benched Embankment in Fault Gouge</u>

A 200-foot high sliver fill was proposed in this area using conventional embankment design. But a field examination indicated that a special foundation design was necessary. The alignment was to cross a merging fault system that not only had a thick soil-talus accumulation on the footwall but severely fractured bedrock and associated gouge that indicated drag along the sheared surface. A standard embankment design would be in jeopardy and would endanger the White Bird Elementary School immediately below.

For these reasons, a series of benches 20 to 25 feet wide, parallel to contour lines were excavated (See Figure No. 1) into bedrock in hopes of removing any potential slip surface, thereby keying the embankment into solid rock. At the embankment catch point, a larger bench was excavated into bedrock which would eliminate any potential base or toe failure.

To date this embankment has not experienced any movement or settlement.

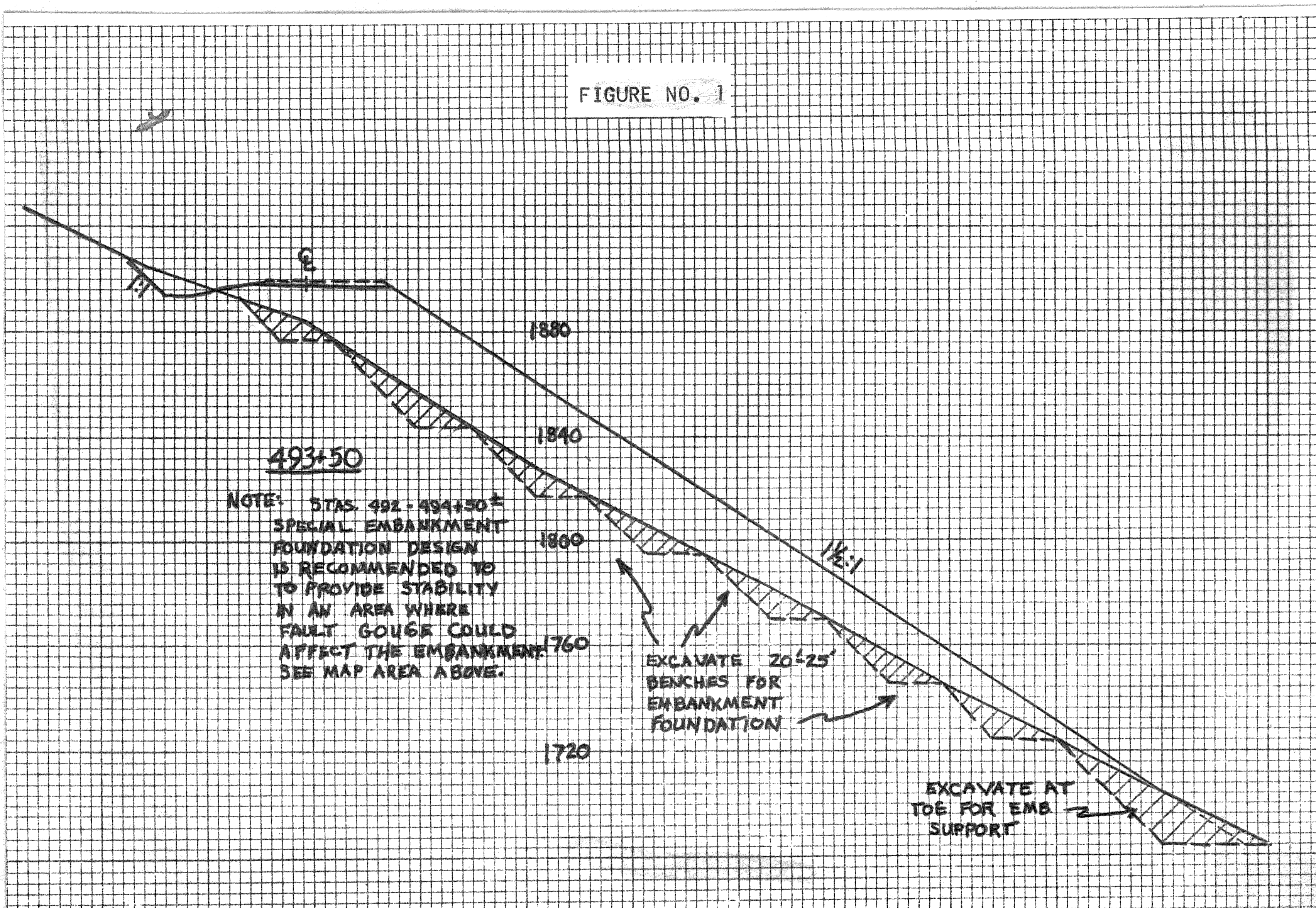
Station 502 - Centerline Fault vs. Slope Design

The influence of the "centerline fault" is shown clearly throughout the vicinity of Station 502. (See Photo No. 3). A number of seismic traverses, both parallel and perpendicular to contour lines were set up in hopes of defining the limits of gouge zone that would affect rock competency within the proposed cut slope.

Although more resistant rock was encountered on the hanging wall, the proposed alignment would not allow one to take advantage of it as it was a vertical outcrop. To do so would necessitate alteration of the curvature which would defeat the purpose, increasing both embankment and excavation. Therefore, it was decided to create the following design which was based entirely on geophysical data.

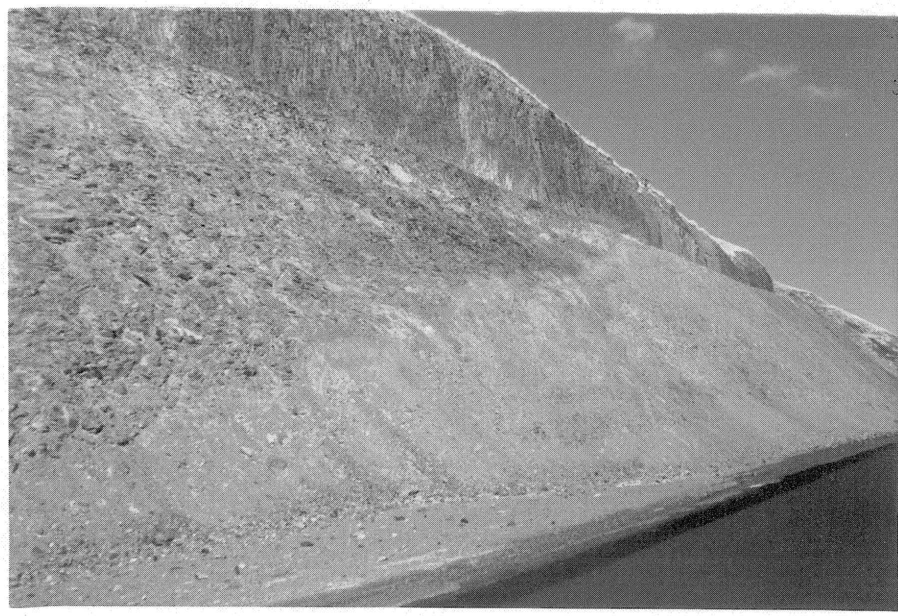
Even though it looks somewhat unusual, Photo No. 3 will indicate that the analysis was correct. The highly fractured loose gouge material

FIGURE NO. 1




Toe Trenching and Serrations for Embankment Stabilization at Sta. 493+50

PHOTO NO. 3



M.P. 224.5
Constructed Backslope and Associated
Fault Gouge at Sta. 501

encountered at grade was designed on a 1:1 slope. It has remained in place with only minimal sluff. The high angle, down the valley fault plane, is exposed throughout much of the upper section. Slopes of 1/8:1 were proposed with 15' wide bench system to catch and retain falling rock. The slope has remained stable as the wall rock behind the shear plane is quite resistant to movement. (See Photo Nos. 3 and 4).

Station 590 - Multiple Faults vs. Slope Design

A deep through cut was proposed in this area, (Photo No. 5), which again raised the question as to what would be a stable, maintenance free slope, yet not be prohibitive cost-wise.

After attempting some detailed mapping, a high angle normal center-line fault was delineated along with some other structural complications that were not readily decipherable. Seismic was attempted but without

PHOTO NO. 4

Aerial View Looking South at New Alignment at Town of Whitebird

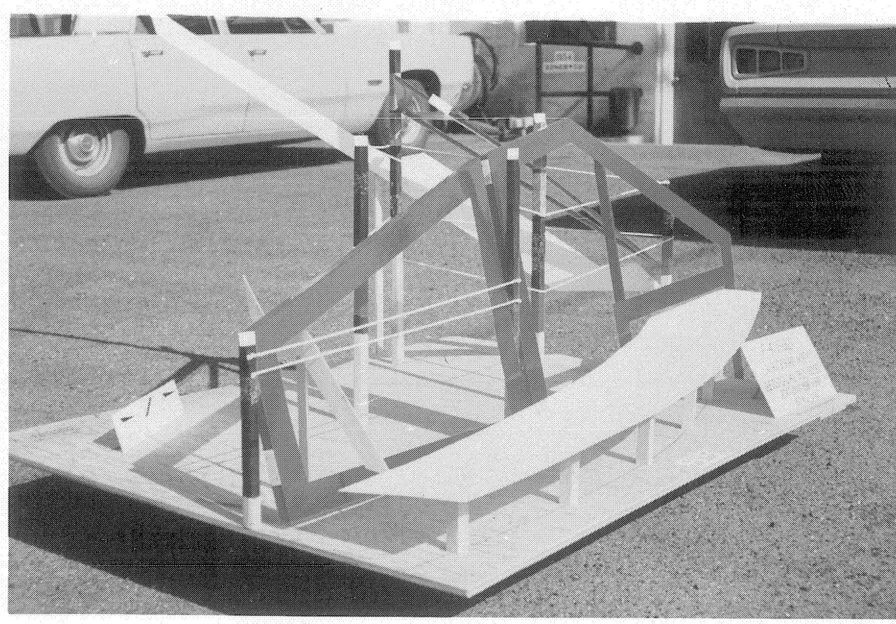
PHOTO NO. 5

M.P. 226.2

Sta. 590. Drill Investigation and Fault Limits much success. Significant amounts of dynamite were used as an energy source (60 percent Gel. 1-1/8" Dia.), but most of the shock was being absorbed in the shear material. Second cycle arrivals were common.

Six diamond drill holes were placed throughout the cut in an attempt to find uniformity or similarity of strata. After the holes were logged a three-dimensional scale model was built depicting the drill holes in relation to the proposed alignment and grade. Then, an effort was made to figure out the structure and amount of throw by correlating the test holes. See Photo No. 6 and Figure No. 2.

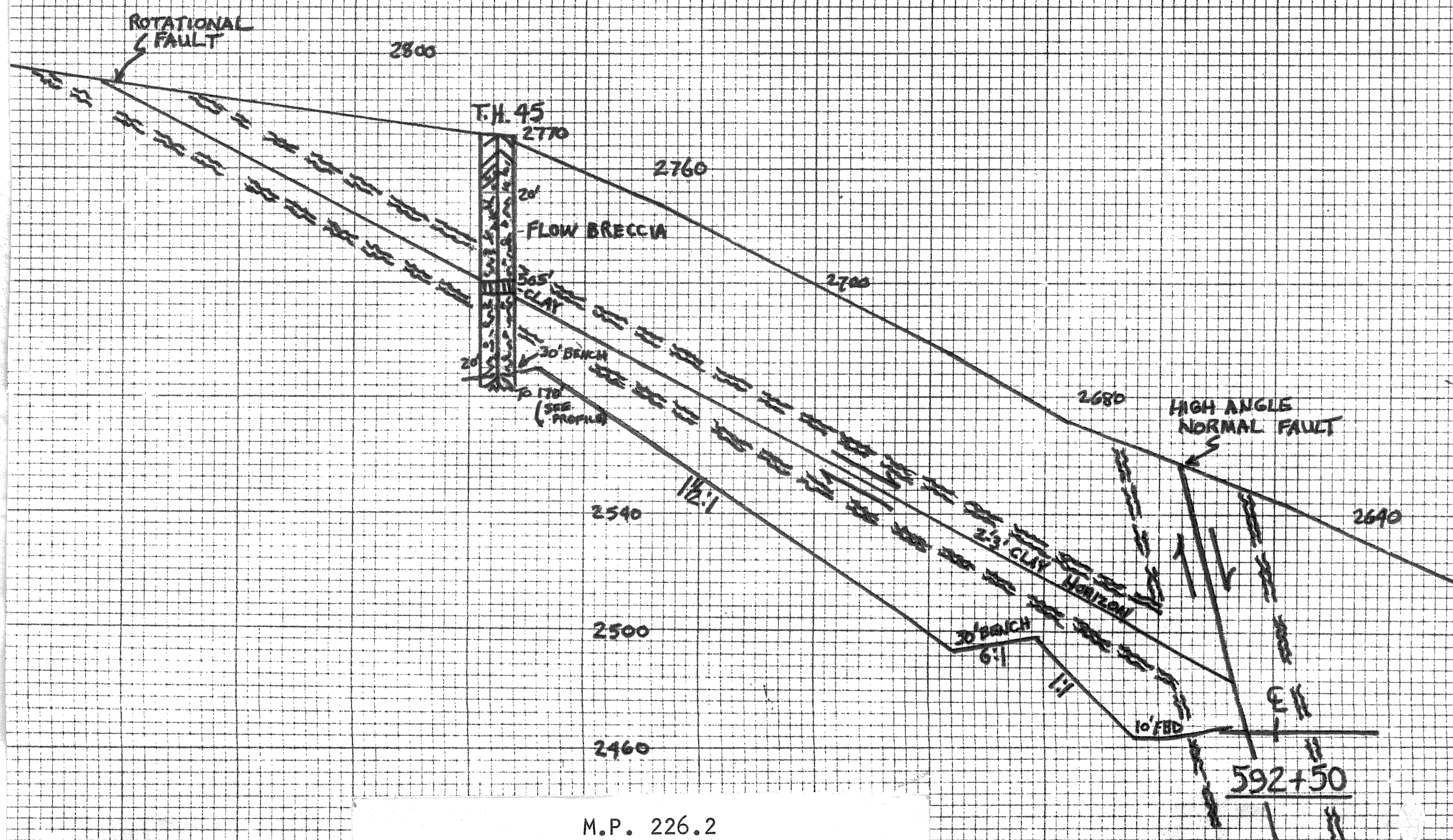
PHOTO NO. 6



M.P. 226.2
Scale Model of Subsurface Conditions and Associated Fault Planes

It appeared that a low angle fault intersected the high fault left of centerline. The cuttings and core indicated highly fractured and altered basalt. A variable slope (1:1 to $1\frac{1}{2}$:1) and bench was designed to eliminate the loading above a three-foot clay horizon in the low angle fault. See

FIGURE NO. 2




M.P. 226.2
Proposed Backslope at Sta. 592+50

Photo No. 7.

PHOTO NO. 7

Constructed Backslope at Sta. 590

Approximately one million yards of material was removed from this cut, with this design. There has not been any subsequent movement or sluff. There of course, has been some criticism, saying the cut was over-designed, primarily because it has not failed.

Station 615 - Settlement Situation

This is an area which is in process of settling. The question is, why? No geologic hazards were present and no problems were expected.

In July 1972 tension cracks (1 inch to 2 inches across) appeared in the base material somewhat paralleling centerline. They more or less followed the projected natural ground line for 200 feet and then disappeared into the embankment. This was thought to be a minor adjustment and ignored.

The project was paved and along in July 1974 the tension cracks reappeared in the same location and of the same magnitude. More concern was generated, especially after an article appeared in the local newspaper.

An examination of the mass diagram and talks with the Resident Engineer showed that the bulk of the embankment material was derived from a degrading "Lower" basalt unit that upon exposure to the atmosphere began to crumble, eventually turning to sand.

Station 670 - Embankment Failure (Postulated)

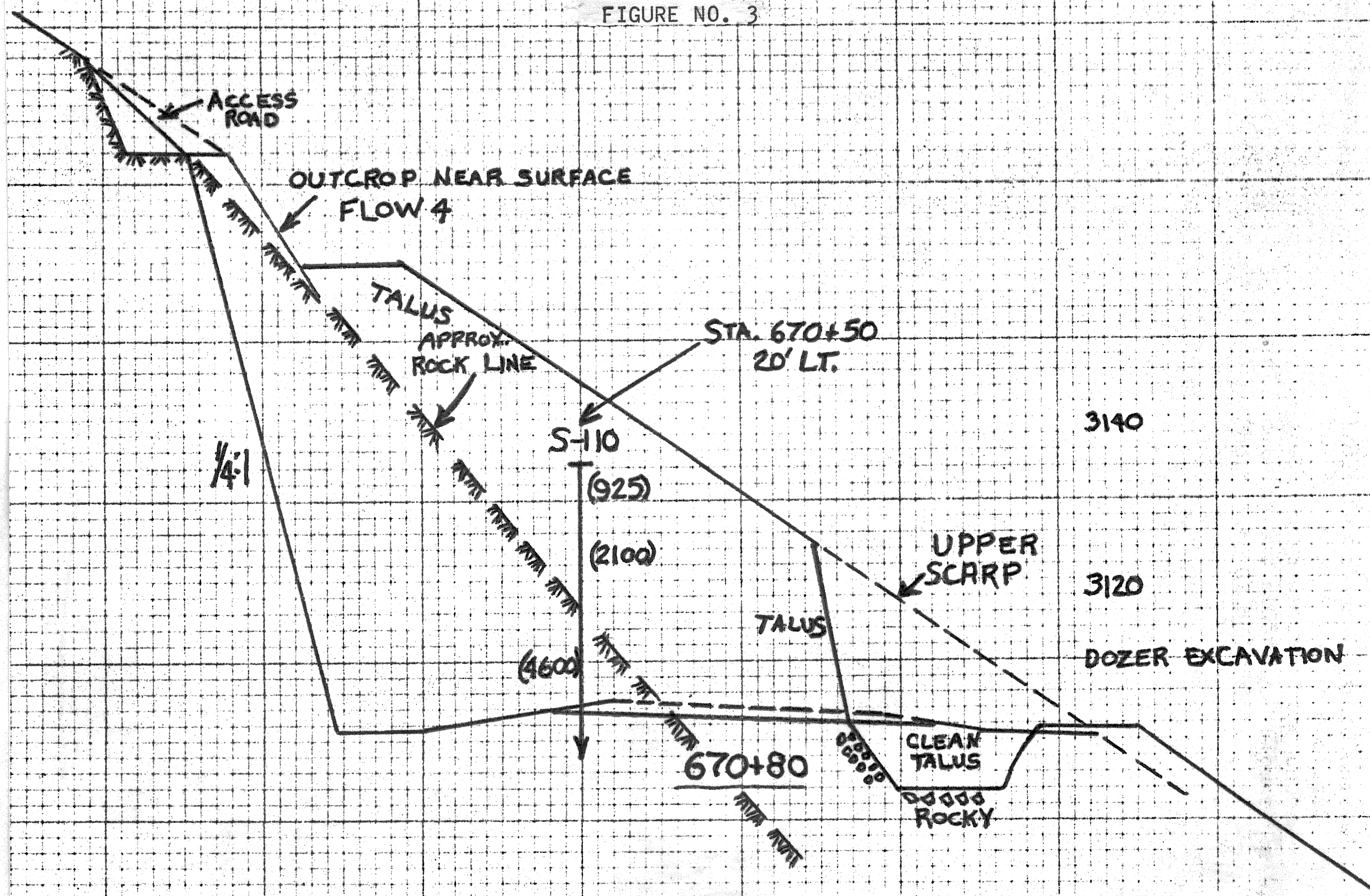
This area, (Figure No. 3) was defined as potentially unstable as existing tension cracks were present on the natural hillside. Deep talus was resting unconformably on a soil terrace. If failure occurred the two outer lanes of the proposed highway would be displaced. Seismic traverses were used to delineate the depth of the soil and talus and rock line (Figure No. 3). A dozer trench was placed to determine the amount of variation within the talus deposit, and if any thin, but detrimental clay or ash horizon was present. None was found.

Rather than shift the alignment into the hill, thereby increasing quantities which were already in excess, management decided to take the risk. The plans indicated that no wasting or side casting would be allowed in this area. Unfortunately, the Contractor was not aware of this and began to end dump. Although he was stopped the damage had been done.

The first indication occurred during November 1970 (Photo No. 8) in which an arcuate scarp had developed along the talus-bedrock interface. By January 1971 the vertical drop was between one and two feet. Fourteen months later the vertical drop was over eight feet in places and had extended into the diced bedrock, (Photo No. 9).

At this time it was decided some corrective action was necessary as the slip plane was continuing to grow and new tension cracks were forming.

FIGURE NO. 3




M.P. 227.7
Sta. 670. Slope Design and Embankment Section

PHOTO NO. 8

M.P. 227.7

Sta. 670. Failure Plan "E" Developing After Wasting on Hanging Terrace

PHOTO NO. 9

M.P. 227.7

Sta. 670. 14 Months After End Dumping Occurred

The most feasible solution was to move into the hillside unless solid rock was situated at a reasonably shallow depth which would allow for a retaining structure.

Due to the nature of the material, coring and tricone drilling were out of the question. It was therefore decided to hire an air track and record the color of the cuttings and drill rate through the talus and bedrock. Twenty-eight holes (952 linear feet) and $1,008.00 later, Materials determined that the amount of variation in respect to rock line was due to a highly undulating pillow-plagonite unit. Also, a significant difference in the potential of the equipment was shown. A more shallow, competent rock line was indicated with the State drill (250 compressor) vs. a rented MR track (500 compressor).

Too much variation existed in the rock line for either the seismic to prove valuable or for the operator to interpret accurately. Due to the variable rock line it was decided to shift the alignment into the hillside and reslope. During July 1975 it was noted the same tension crack has reappeared with approximately 2.0' of vertical drop. It is fortunate the alignment was shifted into the hill.

Station 696+10 to 704+45 - Retaining Structure vs. Sliver Fill

During the final design stages of this project (November 1968) there were two drainages which were still in question. These were sharp, steep, confined gulleys with minimal soil cover. The only water present was during spring runoff and this was negligible.

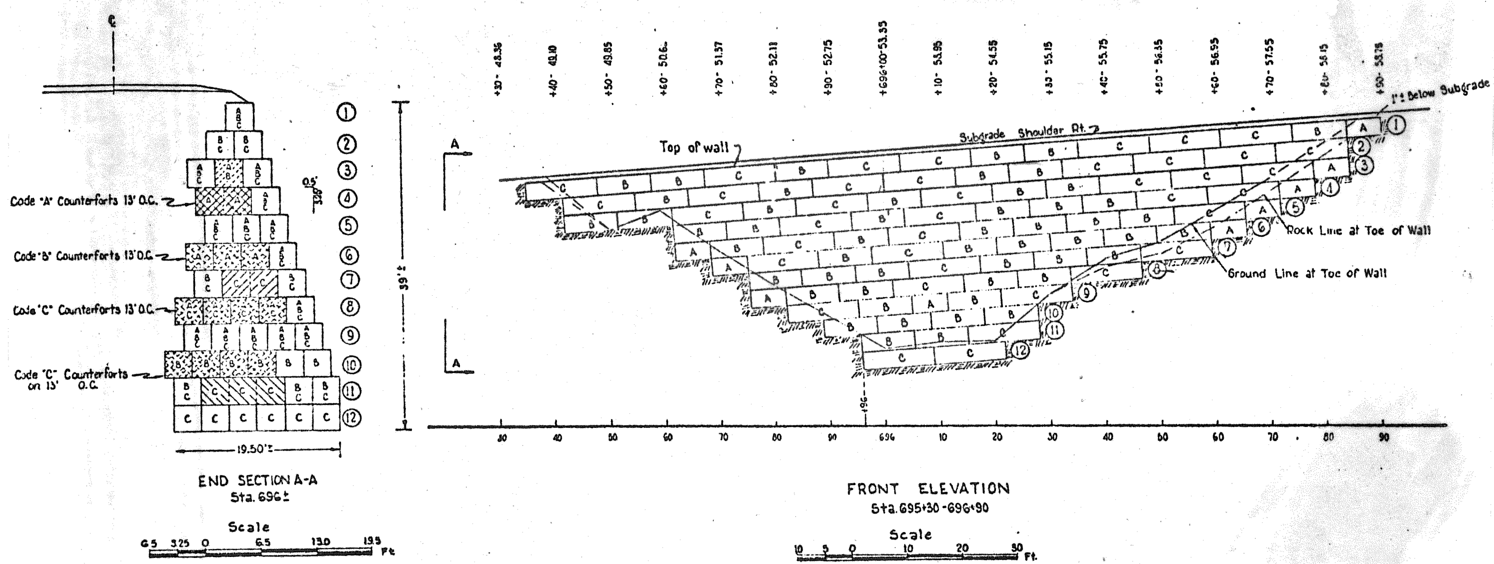
The question was whether to extend a sliver embankment down over the hillside some 280 feet vertically, that could be displaced by a major valley fault, or use some type of retaining structure that would stand 39 feet high and some 160 feet long at grade.

After a cost analysis and review of various types of retaining walls, sales promotions, and a lot of inter-department hassling, Gabion-type bin walls were the answer. The cost estimate for Gabions at that time was approximately half that of the conventional metal bin wall, which the Department was later to find out why. The rock baskets could be easily adapted and keyed to the steep walls of the drainage and it would be ideal to use native basalt. No haul or crushing costs were involved as the quarry rock for rock cap and plant mix was only 200 feet away. The following sequence of Figure Nos. 4 and 5 and Photo No. 10 show the construction procedure along with the end product - failure.

After the failure a backhoe was used to paw around to see where the collapse had occurred. Field examination showed that Course No. 7 of the open rock crib began to bulge taking the front row of Course No. 8 with it. The counter-forts of Course No. 8 were still in place. This was an internal failure due not only to an inadequate design but also to construction procedures. Don't use an open crib design on a relatively high wall. Colorado Highway Department has experienced similar results (Personal Communication).

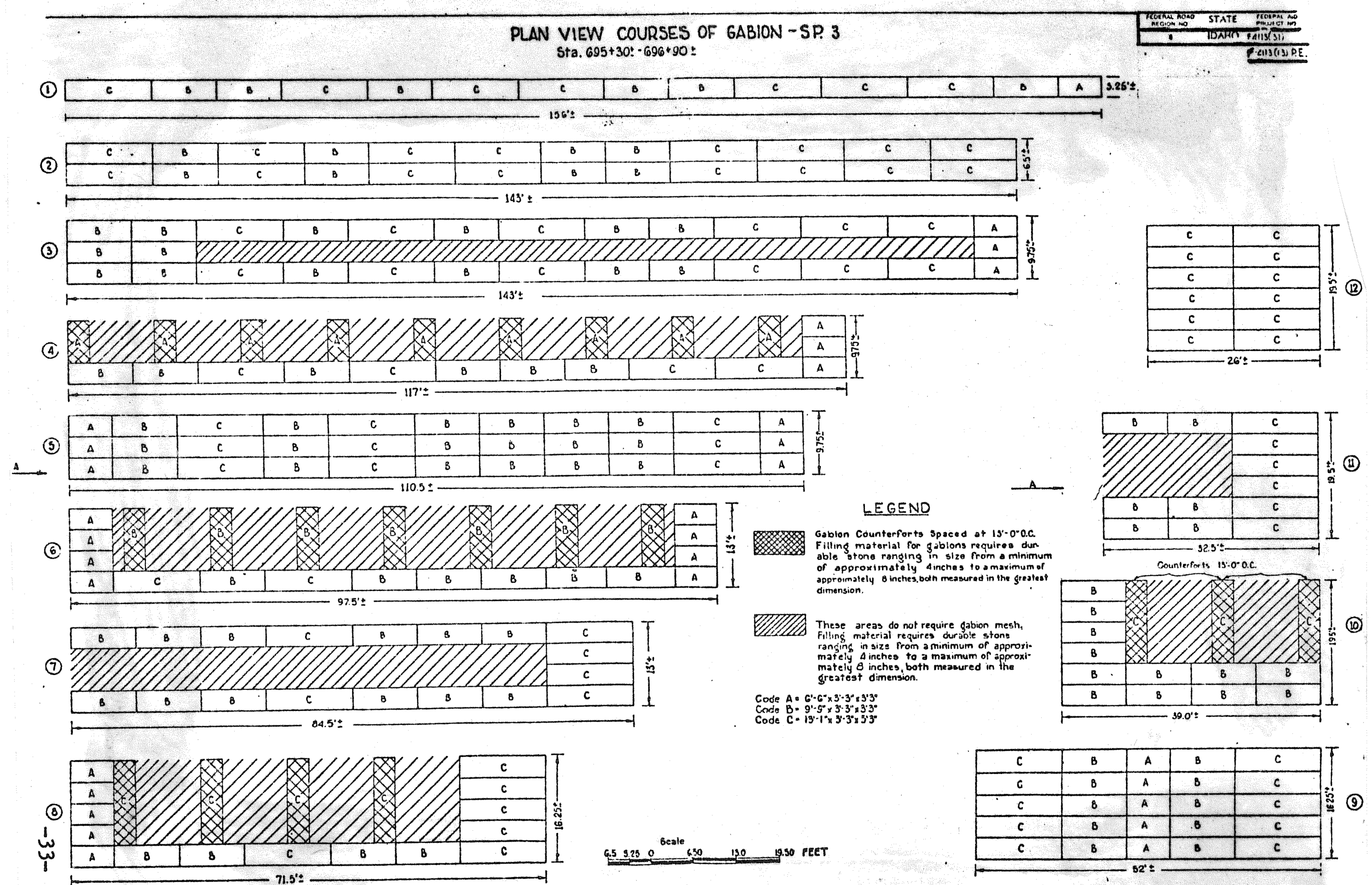
After the failure the design criteria from the Manufacturer changed radically, which increased the cost roughly 100 percent. This included turning the baskets 90° to centerline which increased the rock quantity by nearly double. The twist ties to hold the baskets together were now obsolete. The Manufacturer recommended a spiral lacing procedure. Also the front row of rock facing in the cribs which is the outer wall surface should be hand-placed to insure total stability. A metal bin wall was put

FIGURE NO. 4




M.P. 228.2
Plan Sheet of Proposed Gabion Retaining Wall

FIGURE NO. 5




M.P. 228.2
Internal Counterfort Design of Gabion Wall

PHOTO NO. 10

M.P. 228.2
Gabion Failure

in as the replacement and has performed very well to date.

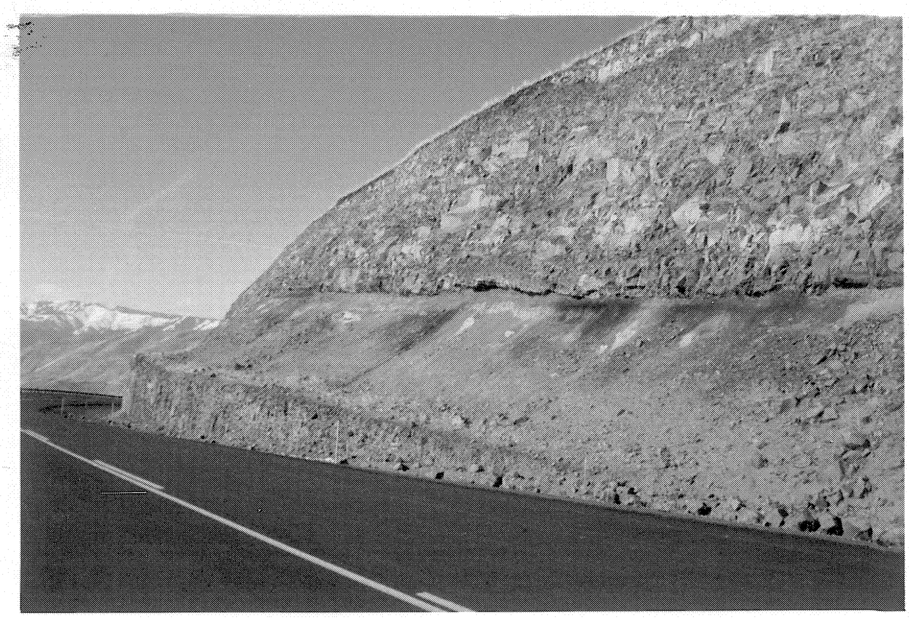
Station 763 - Interbed Treatment vs. Slope and Bench Design

A five-foot to ten-foot thick silty interbed sandwiched between a tightly jointed, vesicular unit below and a more loosely jointed columnar unit above was the subject of a special investigation. Fortunately the interbed was dry, which meant only spalling and differential weathering would have to be considered.

The interbed was initially discovered in the bottom of a diamond drill hole. Once it was known to exist it could be extrapolated along the regional dip to adjacent cut sections. In this instance the only way seismic could delineate the interbed was to attempt up and down slope traverses, which would be plotted to the ground line of a cross section. This was done successfully on later projects where there was minimal overburden to contend with.

Once the thickness and characteristics of the interbed had been determined, benching was the next consideration as the height of the cut was considerable. It was finally determined that benching below the interbed, within the tightly jointed, diced basalt unit would be the most satisfactory. This was determined by seismic analysis as the velocity in this unit was nearly twice as much as the overlying unit, even though the rock quality was the same, Photo No. 11.

PHOTO NO. 11



M.P. 229.20
Sta. 757 + 50. Constructed Backslope

During construction, it was discovered that a small fault (30' through) intersected the backslope which meant that the bench was composed of interbed material in part rather than in the diced unit as proposed. A post construction examination shows that the outer 3.0' of the bench is in danger as tension cracks are present. The interbed has also undercut the overlying columnar unit which has caused minor rock fall onto the bench.

GEOLOGIC MAPPING AND SUBSURFACE INVESTIGATIONS

Sound geologic reconnaissance mapping is essential to insure the successful and timely completion of any project. The answers needed to solve alignment problems may not necessarily be confined to the project but scattered throughout the geologic province. Since this may be miles off, the project management may take a dim view of such wanderings.

To understand the mechanics of the province will add greater insight into the detailed investigation in respect to drill hole placement and geophysical traverses.

The $20,000 spent in punching a dozer road not only gave easy access to the project but also created new exposures that helped define the type and quality of rock. Even with this method such units as the pillow-plagonite member and minor fault displacements, and vent areas were completely missed.

Also related is the noticeable change in the method of investigation. Prior to the White Bird projects discussed in this paper, drilling was the only method of subsurface investigation employed by the Division of Highways. This was not only expensive but also very time consuming. Such an example is the White Bird summit cut in which 31 diamond borings (4481 linear feet) were made for 2300 feet of new road construction, in order to determine the slope design. The F-4113(31) project (18,800 L.F. in length) utilized 28 diamond drill holes (1705 L.F. of drilling) and supplemental geophysical survey techniques. Project F-4113(32) (15,800 L.F. in length) relied predominately on seismic traverses with only 15 diamond drill holes (1056 L.F. of drilling). Since seismic surveys were initiated the number of backslope failures has decreased, along with both cost of investigation and amount of time spent in determining the final design.

## DESIGN STANDARDS IN RELATION TO COST AND GEOLOGIC CONDITIONS

One factor which always contributes to approving a project is the total price tag. The Design Section, along with Management, are usually looking for ways to decrease total cost. The normal way, on projects of this magnitude, is to balance the excavation-embankment quantities. This may be accomplished in several ways: changing grade, changing alignment, or change of backslope design.

Unless these proposed considerations take into account the geologic environment and a slope of adequate safety, problems may develop as occurred at Station 670. The alignment was shifted to reduce quantities and excavation costs, but placed the line in a precarious position. The remedial measures after failure far exceeded the original savings shown by the line shift.

The problem is that geologic risk is difficult to assess in engineering terminology. There are no figures or values a geologist can rely upon to feed a computer that would indicate the total magnitude of risk involved. It is a relative, educated guess indicating a potentially unstable area which may fail tomorrow, in ten years, or never.

Management should be more attentive or in tune with geologic recommendations and not ignore or discard simply because no equation exists for a set of circumstances. The geologists must be able to convey the potential risk in terms that Management can understand.

The White Bird projects proved to be a real challenge from the quantity standpoint, especially figuring the amount of shrink-swell of the various stratigraphic units, as some of these had not been encountered previously. The ability to predict shrink-swell ratios is based more on the experience of the personnel making the prediction rather than anything else. The F-4113(31) and (32) projects had significant overruns in excavation and embankment quantities due to erroneous estimates based on this lack of experience. In the past seven years however, designing and building highways has increased at a much faster pace than before which has allowed the personnel to gain the necessary expertise. The F-4113(38) project along with later projects have been within 5% or less of the actual shrink and swell of the excavation quantities.

## CULVERT PLACEMENT

All cross culverts were originally set up for placement in the bottom of the draws. These foundations had been examined in detail for settlement and compaction problems. Corrugated metal pipes were set up which varied from 18" diameter to 1 gauge multiplate types.

Unfortunately, during construction both the skew angle, location, and gradient of these culverts were changed without the consent or knowledge of Materials or Design Sections. Only the multi-plates remained in the bottom of the draws.

The smaller cross drains were raised, so the outlet end was only a few feet below sub-grade. This meant the runoff would spill directly onto the newly constructed embankment which was part soil and part rock. Deep erosion channels were carved after the first cloudburst. The down-cutting was especially severe throughout the waste areas.

Not only was erosion a problem, but also separation of the pipe joint between the compacted versus the non-compacted section.

Since there was an abundance of waste on the (31) and (32) projects, the uphill side of many embankments were filled. The culverts situated in the embankments, did not flow water, but remained high and dry, as they were not extended to the "V" of the draw which is where the water is concentrated. This water would instead seep into the fill, seeking its own level and hopefully will pass through the compacted embankment without causing any damage.

The solution is easy--end the culverts into the draws. If the gradient is too severe, add dissipaters and splash pads on the outlet end to check erosion. In future projects at least keep the inlet end in the drainage at natural ground.

## BENCH DESIGN

Bench width designed to alleviate rock fall conditions from the finished driving surface should in no instance be narrower than 15'. Anything less gives no margin for error, especially since both contacts and rock quantity may change drastically over a short distance. This width recommendation would be dependent on the backslope design above the bench and prevailing rock conditions. This may be an unknown in many instances until construction begins, especially if the cut was designed or extrapolated seismic data.

The question then asked is whether to design a slope near vertical with a wide ditch and let the post construction cost of rock fall become a maintenance item, or flatten the slopes, and increase quantities of excavation under the existing contract.

The Resident Engineer has the option to make any changes he feels are needed, but unfortunately the Resident is not the most qualified to analyze the situation. There should at least be a consultation with District Materials Personnel on any change either in slope or bench design, as it may be a critical item.

## PRESPLIT CONCLUSIONS

A number of changes should be included within the Division of Highways specifications.

In my opinion, the use of stemming which is the backfilling of borings along the neat line with gravel or rock chips after the explosives have been placed would possibly eliminate the scalloping and over-break in high velocity, resistant flow units where the jointing and fracture habit is only nominally developed.

Even though this would be more expensive, it could alleviate the need for scaling and produce an aesthetically appealing slope with less maintenance. The increased cost of stemming would be offset by the decreased scaling cost.

Experience and accumulated knowledge of rock type and conditions seem to dictate between success and failure of a slope, rather than hard and fast rules. One way to insure this would be to pay for only the footage which shows a percentage of undisturbed hole trace left in the neat line of the backslope. The contractor would not be reimbursed for holes which wandered or drifted beyond the specified tolerances.

DEGRADING LOWER BASALT UNITS

Unfortunately these previously mentioned acceptance tests are applied only to rock which is being considered for aggregate. There are no tests for excavated rock which will be compacted in a fill section.

Basalt from these degrading flow units initially broke out in 2 to 4 foot chunks, many of which were used as barrier rocks. Within one year 66% of these had experienced cracking, separation along joints, and decay into mounds of granular debris.

There is no reason not to expect the same phenomena to occur within the embankment, similar to the effect of an hour glass passing sand from the top through a constriction to concentrate at the bottom. With a calculated swell of 35% from adjacent degrading cuts, voids were built into the embankment. Atmospheric exposure and moisture would initiate the swelling sifting process, that eventually causes settlement.

These flows must be recognized as degrading units and be treated accordingly. The material itself is perfectly acceptable to use in embankments but rather than using the standard grid roller for rock placement, a vibratory roller should be substituted with an adequate amount of water.

This would accelerate breakdown to a sandy component while the embankment is being constructed eliminating most of the voids so the fill would not experience adverse settlement at later date.

The F.H.W.A. has recommended using benzidine dihydrochloride as a staining agent in order to detect the presence of swelling clays which may be present within basalts, especially lower basalts. This chemical, soluable in water and alcohol, may be applied to any hand specimen in the field if it is suspected to degrade. It will immediately turn blue if montmorillonite is present. This procedure should be incorporated into the investigative procedures used by the Department rather than relying on visual indications of deterioration. The geologist could and should test all stratigraphic units during the Phase II report, and set up special handling, placement and rolling for this material in the Phase V Special Provisions.

The Corp of Engineers also indicates that the application of D.S.M.O. causes a more complete breakdown than Ethylene Glycol.

## SEISMIC EVALUATION

This single channel method of investigation for deep cut analysis is slow and consumes a 3-man crew, especially if explosives are used, since charges must be set at each station in order to secure a point on the graph which automatically doubles if the traverse is to be reversed.

If the charge is not heavy enough, second cycle arrivals are received which means the station must be re-shot. If too much powder is used too close to the geophone, a lower threshold or plateau level is achieved which sometimes makes it difficult to pick contacts and blend the data into the preceding hammer line traverse. It was noted that explosives will normally duplicate the hammered slope. A differential in arrival time of 1 to 2 milliseconds is normal due to increased energy.

A charge backfilled by soil, will usually produce better first arrivals than a charge packed with loose rocks.

Traverses which extended across fault gouge zones need additional energy to develop first arrivals, usually 3 to 4 times as much as compared to a line of comparable length within unaltered rock.

## SETTLEMENT OF LARGE FILLS

The greatest single factor governing the performance of a fill section appears to be time. The paving contract for the F-4113(31) and F-4113(32) projects were let in 1973 or about 2 years after completion of the embankments. During this period the fill sections had time to set and adjust. The vibration of haul traffic created by construction equipment produced additional settlement. Remeasurement, for the subsequent paving contract, determined that approximately 2.0' of settlement had occurred in the preceding period. (Station 773+50, depth of embankment 150' at centerline).

It therefore is apparent that the embankment settlement time factor becomes important for normal construction procedures. It is suggested that the use of vibratory rollers and adequate amounts of water will reduce this time factor required for settlement and produce a more tolerable limit.

## RAPPORT WITH HIGHWAY CONSTRUCTION PERSONNEL

It is imperative that the Materials Section acquaint the Resident Engineer and Project Engineer with the soils-geophysical profile (Phase II Report) as there is a wealth of information that can be utilized during the construction of the project. Also, accompanying the profile is a topographic-geologic map with the various engineering units noted and described, with unstable or hazardous conditions noted. The alignment should be reviewed, with the Resident Engineer, in the field, at the time the Special Provisions are being drafted to see if he concurs with the recommendations proposed or can specify a more feasible approach.

After the project is awarded the Resident Engineer, faced with a field problem or change which will affect the previous recommendations made by the Materials Section, should consult with that Section.

Post-construction failures and problems could be alleviated by this coordination.

CONCLUSIONS - SEISMIC

1. Fault gouge may be minimal and not show a normal seismic peak. This could depend upon velocity of the adjacent flows.

2. In basalts, low velocity is a function of jointing, not necessarily of the rock quality.

3. Don't assume vertical contacts on seismic data in basalt sections that are covered with varying amounts of soil-colluvium-talus.

4. Faults-gouge and interbeds can be delineated if the stratigraphy and geologic history are understood. Lack of field time and basic mapping leads to more interpretation mistakes.

5. In areas of rock fall vs. slope design, correlation must be made from existing exposures and seismic velocities.

6. Velocity range, less crucial at higher velocities, but more room for error in flatter slope interpretations.

7. A gray area in basalts, 2500 to 4000'/sec. may indicate pillow lava, shear zones, altered or tight interlocking talus, moisture, flow top and soil. Additional investigation is warranted in these questionable areas.

8. Low velocities not always indicative of ripping vs. presplitting. Pillow lavas and breccia, due to lack of prominent jointing, can be presplit economically.

9. Contractors have found geophysical information extremely useful in bidding the job and using the information for presplitting and produc-

tion round blasting vs. ripping operations, which generally means a better construction price.

10. It must be remembered that a seismograph is only a supplemental tool of your exploration program and must be coordinated and calibrated to existing norms.